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August 23, 2013
Kleinfelder Project No: 135118

Ms. Carla Muscarella via e-mail (CMuscarella@Manhard.com)
Manhard Consulting Ltd
3300 Highlands Parkway, Suite 255
Smyma, GA 30082

SUBJECT: Geotechnical Engineering Report
Culvert Replacement
Martin Slough – Highway 395
Douglas County, Nevada

Dear Ms. Muscarella:

Kleinfelder is pleased to present the results of our geotechnical engineering assessment performed for the proposed removal and replacement of an existing culvert crossing underneath Highway (Hwy) 395 at the Martin Slough, north of Minden in Douglas County, Nevada. This report includes background information regarding the anticipated construction, the purpose and scope of services provided, discussions regarding the investigative procedures, and the site conditions encountered during the field exploration. Geotechnical conclusions and recommendations are provided for project design and construction. The appendices of the report include logs of borings and a summary of laboratory tests. Also included is an information sheet published by the American Society of Foundation Engineers (ASFE). Kleinfelder is a member of ASFE, and we feel this sheet will help you better understand geotechnical engineering reports.

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We appreciate this opportunity to be of service to you, and look forward to future endeavors. If you have any questions regarding this report or need additional information or services, please feel free to call one of the undersigned in our Reno office.

Sincerely,

KLEINFELDER

Kyle Lenehan
Staff Engineer

PROFESSIONAL ENGINEER - STATE OF NEVADA
JONATHAN W. PEASE
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1. INTRODUCTION

1.1 GENERAL

This report presents the results of Kleinfelder's geotechnical engineering study for the planned removal and replacement of an existing box culvert crossing underneath Highway (Hwy) 395 at the Martin Slough, north of Minden in Douglas County, Nevada. The location of the project site is shown on Plate 1, Site Vicinity Map. Our services for this study were performed in accordance with Kleinfelder's proposal dated April 22, 2013.

This report includes our recommendations relating to the geotechnical aspects of project design and construction. The conclusions and recommendations stated in this report are based on the subsurface conditions found at the locations of the soil borings at the time our exploration was performed. They also are subject to the limitations and provisions stated in Section 5 of this report.

1.2 PROJECT DESCRIPTION

We understand that the proposed project consists of removing existing dual 3- by 6-foot (height by width) box culverts and replacing them with two 4- by 12-foot box culverts. We have assumed the new box culvert will be constructed using pre-cast box culvert sections. The length of the new culvert is approximately 163 feet. The new box culvert will be founded at approximate elevation 4,694 feet, and loads for the new box culvert are anticipated to be similar to the current loads of the existing box culvert. Cuts and fills are not proposed and the final grade will match the existing grade.

Wingwalls up to approximately 5 feet in height are planned at the inlet and outlet of the box culvert. Kleinfelder understands the wingwalls will be supported on continuous spread foundations. Structural loads were assumed to be less than 2 kips per lineal foot.

If the structural loads or descriptions are different from those described above, Kleinfelder should be notified to re-evaluate the recommendations contained in this report.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore and evaluate the subsurface conditions at the project site in order to develop recommendations related to the geotechnical aspects of project design and construction.

The scope of services, as outlined in Kleinfelder's proposal, dated April 22, 2013, included:

- A visual site reconnaissance of surface conditions;
- A field investigation that consisted of drilling borings within the area of the proposed development to explore the subsurface conditions;
- Laboratory testing of representative samples obtained during the field investigation to evaluate relevant physical and engineering parameters of the subsurface soils;
- Evaluation of the data obtained and an engineering analysis to develop geotechnical conclusions and recommendations; and
- Preparation of this report which includes:

    - A vicinity map/boring location plan
    - A description of the proposed project
    - A description of the field and laboratory investigations
    - A description of the surface and subsurface conditions encountered during the field investigation
    - Conclusions and recommendations related to the geotechnical aspects of the project design and construction, including seismic parameters, general earthwork, foundation design, and lateral earth pressures for design retaining structures
    - Appendices that include logs of borings and a summary of laboratory tests.

This study does not include detailed investigations of site-specific seismicity, faulting, or other geologic hazards.

2 FIELD AND LABORATORY EVALUATIONS

1.4 FIELD EXPLORATION

Kleinfelder originally proposed to drill three soil borings at the site; on the east side, in the median, and on the west side of Hwy 395. On July 25th, 2013, Kleinfelder's field engineer evaluated the site once underground utilities had been marked by various agencies. The engineer determined that a safe and suitable drilling location could not be identified on the west side of Hwy 395 due to the high volume of underground utilities. The median of Hwy 395 was missing obvious utility markings and was also deemed unsafe to drill due to the likely presence of unmarked utilities. Two locations were identified on the east side of Hwy 395 and soil borings were advanced to assess some of the potential soil variability on the site. Kleinfelder immediately notified Manhard of the field conditions; it was agreed that there should be further explorations at that time and the drill rig was demobilized from the site. Following the completion of the field exploration, Kleinfelder delivered a summary of the field activities to Manhard to present to the owner, Douglas County. It is our understanding that Douglas County decided to not remobilize and drill additional borings. The two explorations on the east side of Hwy 395 were advanced to approximately 9 and 20 feet below ground surface. Both borings encountered practical refusal due to heaving sands. Approximate boring locations are shown on the Exploration Location Map (Plate 2).

The exploratory borings were advanced using a truck-mounted drill rig equipped with 6-inch outside diameter (O.D.) hollow-stem augers. Subsurface soil samples were obtained during exploration using a standard split-spoon sampler (2.0-inch O.D.) and a "California" split-spoon sampler (3.0-inch O.D.) driven with blows from a 140-pound safety hammer falling through a 30-inch drop. The blows required to advance the split-spoon sampler in 6-inch increments below the bottom of the borehole are recorded on the logs. These blow counts are an indication of the relative density or consistency of the strata.

The logs indicate the type of sampler used for each sample. When the sampler was withdrawn from the boring, the liners containing the samples were removed, examined for logging, labeled and sealed to preserve the natural moisture content for laboratory testing. In cohesive soils, our engineer obtained approximate measurement of undrained strength using a pocket penetrometer. Groundwater levels were measured at the end of drilling and before backfilling the boring. Strength measurements and groundwater levels are presented on the boring log.

Soil conditions encountered are presented on the boring logs in Appendix A. The soils were initially identified in the field by a Kleinfelder field engineer in general conformance with ASTM D 2488. The lines defining boundaries between soil types on the logs are based upon Kleinfelder's field observations and interpolation between samples and are therefore approximate. Transition between soil types may be abrupt or may be gradual.

Additional classification was subsequently performed based on laboratory testing in accordance with ASTM D 2487. A description of the Unified Soil Classification System used to identify the site soils and keys to boring log symbols and abbreviations are presented in Appendix A.

Samples obtained during the field exploration were transported to our laboratory for further examination and testing. Samples will be retained for a period of 90 days from the date of this geotechnical report after which time samples will be discarded unless otherwise requested.

1.5 LABORATORY TESTING

Geotechnical laboratory tests were performed on selected soil samples to estimate their relative engineering properties. Testing for the following properties was performed in general accordance with recognized standards:

- Atterberg Limits;
- Moisture-Density;
- Particle Size Analysis; and
- One-dimensional Consolidation;

Results of the geotechnical laboratory tests are included in Appendix B of this report. Selected geotechnical test results are also shown on the boring logs contained in Appendix A.

3 SITE CONDITIONS

3.1. GEOLOGIC SETTING

The site is located in the Carson Valley on the eastern flank of the Sierra Nevada geologic province within the Basin and Range geologic province. The Basin and Range geologic province is broken into a series of north-south-trending valleys and mountain ranges by north-south-trending normal faulting. The Carson Valley is bounded to the west by the Sierra Nevada Mountains and to the east by the Pine Nut Mountains. Most of the displacement on the faults and lineaments within the area occurred during Tertiary time, although earthquake activity continues to the present day in much of the Basin and Range province.

During the Pleistocene epoch glaciers grew in the Sierra highlands and made their way down former stream channels, carving U-shaped valleys and depositing large amounts of sediment into the adjacent basins. During the late Pleistocene to Holocene, between approximately 30,000 and 10,000 years before present, as the glaciers began to recede, widespread lakes and rivers formed in the basins adjacent to these mountains, including in the Carson Valley where the project is located.

A review of the *Preliminary Geologic Map of the Minden Quadrangle, Douglas County, Nevada, and Alpine County, California* (Ramelli, et. al., 2009) indicates the project site is underlain by meander-belt deposits (Qme_{1c}) of the mid-late Holocene, earlier meander-belt deposits (Qme_{1a}) of the late Holocene, and recently active meander-belt deposits (Qmea) of the late Holocene to present.

3.2. SITE CONDITONS

The project site is located on Hwy 395 approximately 800 feet north of the intersection of Hwy 395 and Ironwood Drive. The existing box culvert is located underneath the roadway embankment and conveys water in Martin Slough from the east side of Hwy 395 to the west side. The eastern and western boundaries of the box culvert are located approximately 30 feet beyond the edge of the roadway pavement. At the location of the existing culvert, the top of the roadway embankment is 6 to 7 feet higher than the adjacent grade. The four-lane highway has paved shoulders on both the east and west sides, with widths of 30 and 8 feet respectively, and a median approximately 10 feet wide. Numerous underground utilities were marked on the west side of Hwy 395, running parallel to the highway. At the time of our investigation, the areas surrounding the inlet and outlet of the box culvert consisted of thick shrubby vegetation and occasional trees.

3.3. SUBSURFACE CONDITONS

Subsurface conditions encountered in our explorations were generally consistent with those described in the previously referenced geologic map. The soil profile in boring B-2 generally consisted of roadway embankment fill consisting of silty sand to approximately 6 feet depth or Elevation 4,697. Boring B-1 was drilled in native ground at the toe of the embankment, in which the native soils consisted of soft sandy lean clay and loose clayey sand with low to medium plasticity to approximately 4 to 5 feet depth or Elevation 4,693. It is anticipated that the proposed box culvert will be founded on the soft sandy lean clay and loose clayey sand layer. Underlying the sandy lean clay and clayey sand was 3-5 feet thickness (to Elevation 4,689) of soft lean clay with medium plasticity overlying 2 to 3 feet thickness of dense gravel (to Elevation 4,687). Sands were encountered below the dense gravel layer and were unable to be sampled due to the heaving of the saturated materials; however the sands were assumed to be dense to very dense based on local experience. Kleinfelder has assumed the poorly graded gravels and sands extend to below the total depth explored (to Elevation 4,683).

3.4. GROUNDWATER

Groundwater was encountered approximately 3.5 feet below the approximate top of boring and approximately 7.5 feet below the top of the roadway embankment, at approximate Elevation 4696, 2 feet higher than the proposed culvert invert elevation. Free standing groundwater was initially encountered 5 feet below ground surface in boring B-1. Once the lower permeability clays and fills were penetrated, the free standing groundwater rose to 3.5 feet below the top of the boring. It should be noted that soil moisture levels and groundwater levels commonly vary with time depending upon seasonal precipitation, irrigation practices, land use and runoff conditions.

3.5. GEOLOGIC HAZARDS

3.5.1 Seismicity and Faulting

A review of *Preliminary Geologic Map of the Minden Quadrangle, Douglas County, Nevada, and Alpine County, California* (Ramelli, et. al., 2009) indicates no mapped faults cross or trend towards the project site. A review of the *Quaternary Fault and Fold Database for the United States* by the U.S. Geological Survey and the Nevada Bureau of Mines and Geology (2006), indicates the nearest mapped active faults (indicating fault displacement during the last 10,000 years) are lineaments of the Genoa fault zone and the Eastern Carson Valley fault zone located approximately 3.5 miles west and 2.1 miles east of the project site, respectively. The Genoa fault zone and the Eastern Carson Valley fault zone are estimated to be capable of generating an earthquake of moment magnitude 7.4, and 6.7, respectively. The project site also lies within the zone of influence of numerous other fault systems in western Nevada and eastern California. Should a seismic event occur along any of the nearby faults or fault systems, the site could be significantly affected by ground shaking.

3.5.2 IBC Seismic Design Criteria

The 2009 *International Building Code* (ICC, 2009) requires a detailed soils evaluation to a depth of 100 feet to develop appropriate soils criteria. However, the code states that a Site Class D may be used as a default value when the soil properties are not known in sufficient detail to determine the soil profile type. The Site Class D soil profile is for stiff soils with a shear velocity between 600 and 1,200 feet per second, or with an Standard Penetration Test resistance between 15 and 50 blows per foot, or an undrained shear strength between 1,000 and 2,000 pounds per square foot (psf). Based on our experience and the geology at the site, it is Kleinfelder's opinion that the default Site Class D is appropriate.

The Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1) were estimated using Section 1615 of the 2009 IBC. The mapped acceleration values and associated soil amplification factors (F_a and

F_v) are also presented in the table below. Corresponding site modified (S_{MS} and S_{M1}) and design spectral accelerations (S_{DS} and S_{D1}) are also included.

Table 1 - Ground Motion Parameters Based on 2009 IBC

Parameter	Value	2009 IBC Reference
Approximate Latitude	38.9667	
Approximate Longitude	-119.7790	
S_S	1.68g	Section 1615.5
S_1	0.76g	Section 1613.5
Site Class	D	Table 1615.1.1
F_a	1.00	Table 1615.1.2(1)
F_v	1.50	Table 1615.1.2(2)
S_{MS}	1.68	Section 1615.1.2
S_{M1}	1.14	Section 1615.1.2
S_{DS}	1.12g	Section 1615.1.3
S_{D1}	0.76g	Section 1615.1.3

We note that the site is underlain by potentially liquefiable soils, and therefore technically it should be classified as Site Class F, which requires site-response analysis to develop seismic design parameters. However, there is an exception to this requirement if the fundamental period of the structure is less than 0.5 second per ASCE 7-05 Section 20.3.1. It is our understanding that the fundamental period of this structure is less than 0.5 second. Based on the above, we classified this site as Site Class D.

3.5.3 Liquefaction

A quantitative liquefaction analyses was outside the scope of this investigation and has therefore not been performed. Qualitative discussion is provided below based on Kleinfelder's experience in the area and encountered subsurface material to limited depths.

Liquefaction is a nearly completed loss of soil shear strength that can occur during a seismic event in saturated, loose to medium dense, poorly-graded sands, silty sands, cohesionless silts, and some gravels. Liquefaction results from cyclic shear stresses and strains causing partial collapse of the soil matrix and development of excessive pore water pressure between the soil grains. Liquefaction will result in settlements shortly after the earthquake. Water and sand may be expelled to the surface, referred to as sand boils; these may cause minimal damage, except if footings are located directly over a major sand boil. For sites with gentle or minimal slopes or with an adjacent slope, significant damage may potentially result from ground oscillation or lateral spreading. Uplift can occur to buried structures, such as the proposed box culvert, which are less dense than the surrounding soil.

Given the general geologic conditions, the presence of loose silty to clayey sands and soft fine-grained soils, and the anticipated depth to groundwater at the site, we anticipate the site to be potentially susceptible to liquefaction generally between 5 and 10 feet depth. Potentially liquefiable medium dense sands could be present at a depth greater than the maximum depth explored, however, cannot be determined based on the available data.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1. GENERAL CONCLUSIONS

Based on the results of our field and laboratory investigations, it is Kleinfelder's opinion that this site is generally suitable for the replacement box culvert provided that the recommendations contained in this report are followed. These opinions, conclusions, and recommendations are based on the field explorations, engineering analysis, the properties of the materials encountered in the borings, the results of the laboratory testing program, and our understanding of the proposed development of the site. Where applicable, we assumed that construction methods and materials must meet or exceed Nevada Department of Transportation specifications (NDOT, 2001) and standard plans (NDOT 2007).

4.2. EARTHWORK

4.2.1 Site Clearing and Preparation

Prior to construction, all existing improvements (pavements, curb and gutter, underground utilities, etc.) will need to be demolished and removed from the site or protected in place. Bituminous pavements, concrete curb, and gutters, should be removed to neatly sawed edges. Existing asphalt pavements should be disposed of off site or stockpiled and processed for reuse beneath new pavements.

Areas with existing vegetation should be stripped / grubbed of organic soils, tree roots, etc. Approximately 4 to 6 inches can be used as a reasonable estimate for average depth of stripping of the embankment, but a foot or more will likely be necessary in lowland and channel areas. Deeper stripping/grubbing of organic soils, roots, etc., and potentially removal of debris fill may be required locally. Tree root balls should be removed and the resulting voids backfilled with adequately compacted backfill soil. Dust control should be the responsibility of the Contractor.

4.2.2 Temporary Excavations

The Contractor is responsible for site safety and all excavations should be evaluated to verify their stability, prior to occupation by construction personnel. We do not expect the walls of excavations in the site fill soils to stand near vertical without sloughing. The Contractor should be prepared to shore or slope excavations in these materials.

Excavations will require shoring or laying back of sidewalls to maintain adequate stability. Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) requires that temporary sidewall slopes be no greater than those presented in Table 2.

TABLE 2 – MAXIMUM ALLOWABLE TEMPORARY EXCAVATION SLOPES

Soil or Rock Type	Maximum Allowable Slopes[1] for Deep Excavations less than 20 Feet Deep[2]
Stable Rock	Vertical (90 degrees)
Type A[3]	3H:4V (53 degrees)
Type B	1H:1V (45 degrees)
Type C	3H:2V (34 degrees)
Notes:	
1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.	
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.	
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavation in Type A soils that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).	

The State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health, has adopted and strictly enforces these regulations, including the classification system and the maximum slopes. In general, Type A soils are cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf. Type C soils have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. The client, owner, design engineer, and contractor shall refer to Appendix A and B of Subpart P of the previously referenced Federal Register for complete definitions and requirements on sloping and benching of trench sidewalls. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

For the clayey site soils, trench excavations should comply with current OSHA safety requirements (Federal Register 29 CFR, Part 1926) for Type B soil. In the granular soils, if present, excavations will need to be modified to comply with OSHA requirements for Type C soil. Any area in question should be considered Type C, unless specifically examined by the contractor's engineer during construction. Conditions more restrictive than Class C could result if the contractor does not provide adequate groundwater control. All trenching shall be performed and stabilized in accordance with OSHA standards.

For any temporary slopes, the cut faces should be inspected by the Contractor during the work day for any signs of movement and tension cracks. Workers should not be allowed to work near the excavations unless such inspections deem the area safe. During periods of heavy precipitation or high stream levels, a potential exists for sloughing of the cut slopes and precautions should be taken. In the event the soils become wet from a storm event, or any other source; work should be halted until the stability of the slopes is reassessed.

During wet weather, runoff water should be prevented from entering excavations. Water should be collected and disposed of outside the construction limits. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance of one-third the slope height from the top of any excavation.

4.2.3 Subgrade Preparation and Subgrade Stabilization

Granular roadway fill was estimated to exist approximately to 6 to 7 feet below the center of the highway embankment. Hard to very soft sandy lean clay and loose clayey sand is present below the fill to below the planned box culvert invert, and groundwater is present to above the planned box culvert invert. These soil conditions will require stabilization. Prior to excavation, the groundwater should be sufficiently lowered to prevent unstable conditions. Light, track-mounted construction equipment should be used in excavations to help prevent destabilizing the subgrade soils and causing "pumping". Alternatively, excavations should be performed using equipment from the top of the embankment and heavy construction equipment should not be used in the bottom of excavations. In the event unstable soils are encountered in the excavation bottoms, additional construction dewatering, overexcavation and/or subgrade stabilization would be necessary.

4.2.4 Subgrade Stabilization

For preparation of the box culvert subgrade and for any construction that may be performed during wet weather, the subgrade soils will most likely be well above optimum moisture content and difficult to impossible to compact. In some situations, moisture conditioning of the top 12 inches of subgrade may allow the soil to dry sufficiently to allow compaction. Where construction schedules preclude delays or drying is ineffective, mechanical subgrade stabilization will be necessary. Subgrade stabilization is usually a trial and error process, typically determined with a test section. The final selection of a method of stabilization and final subgrade stabilization is the contractor's responsibility.

For box culvert construction, mechanical stabilization may be achieved by over-excavation and placement of drain rock (Types 1 or 2 drain backfill, Standard Specifications Section 704.03.01). Placement of separation geotextile underneath the rock fill could also be considered, such as Mirafi 180N, Mirafi S800, Geotex 311, or equal. This fill should be densified with small equipment, such as a small self-propelled sheeps-foot roller, until no further deflection is noted. The drain rock bedding would ease minor grade adjustments during RCB placement, and could potentially provide a supplemental drain material for groundwater removal. The separation geotextile must also be placed between the drain rock and any overlying finer fill materials.

If the stabilizing fill is intended to support extensive vehicular traffic, additional geotextile strength and/or fill thickness may be required. The contract documents should provide flexibility for additional subgrade stabilization and overexcavation as needed during grading operations.

4.2.5 Embankment Materials

It is assumed that excavations around the new culvert will be limited in lateral extent and will meet NDOT requirements for structural excavation and granular backfill (Standard Plans Sheet R-1.1.4, Standard Specifications sections 206 and 207). If the existing culvert (to be removed) is offset from the new culvert, it is assumed that excavation shall meet the requirements for structural excavation and backfill (with either onsite roadway excavation or import borrow material) as roadway embankment (Standard Specifications 206 and 203). Granular backfill specified in Section 207.02.02 generally consists of a quarried, crushed sand product. Import borrow should have an R-value of 45 or greater. The existing embankment fill to be excavated should be suitable for reuse as roadway embankment; however, portions of the fill may be above optimum water content and would need to be windrowed and dried offsite.

Native material underlying the embankment fill will not meet NDOT requirements and will be required to be disposed of off-site.

4.2.6 Fill Compaction

All fill soils, either native or imported, required to bring the site to final grade should be compacted as roadway embankment or granular backfill (Standard Specifications Section 207.03.01). The fill should be uniformly moisture conditioned to a moisture content within 2 percent of the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent of the maximum dry density as determined from NDOT test T108. Backfill for wingwalls should be to at least 95% of the maximum dry density. Additional fill lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. Discing and/or blending may be required to uniformly moisture condition soils used for structural fill. In all cases, the finished surface should be smooth, firm, and show no signs of deflection. Grading should not be performed with or on frozen soils.

4.2.7 Utility Trench Backfill

The maximum particle size in trench backfill should be 4 inches. In general, bedding and initial backfill 12 inches over the utility will require import and should conform to the requirements of the utility having jurisdiction. Bedding and initial backfill should be densified to at least 90 percent relative compaction. Native granular soil will provide adequate final backfill as long as oversized particles/debris are excluded, and should be placed in maximum 8-inch-thick loose lifts that are compacted to a minimum of 90 percent relative compaction in all structural areas. Trench backfill within pavement areas, beneath slabs, and adjacent to foundations should be compacted in six- to eight-inch layers with mechanical tampers. Jetting and flooding are not permitted. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement structural section or other overlying improvements.

Excavations below the ground water table (if utilities will be present below about 8 to 10 feet depth) will likely require dewatering. Below the waterline, bedding and backfill should consist of compacted drain rock graded in accordance with the requirements for Types 2 drain backfill presented in the Standard Specifications Section 704.03.01. When drain rock is used as trench backfill, it shall be considered a rock backfill (greater than 30 percent retained on the 3/4-inch sieve) and should be placed in maximum 12-inch-thick loose lifts, with each lift densified by at least five complete passes with approved compaction equipment and until no deflection is observed. A separator geotextile should be placed between the drain rock and any overlying finer-grained soil backfill.

Trench backfill recommendations provided above should be considered minimum requirements only. More-stringent material specifications may be required to fulfill bedding requirements for specific utility types. The project Civil Engineer should develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

4.2.8 Shoring

Shoring will be required where space or other restrictions do not allow a sloped construction slope, and where loose fill material is encountered near the surface, or excavations may be shored at the option of the contractor. Layers of cohesionless sands in the exiting embankment fill material should be anticipated that would slough or ravel and would not retain a vertical cut. Any shoring design (e.g. soil nail, tieback anchor/solider pile, etc.) should consider, among other things: bottom heave/shear failure at/below shoring walls; groundwater inflow in and below shoring; effect of temporary stand-up time in cohesionless soil; flowing sands; presence of cobbles and boulders.

Any shoring system would have to meet OSHA pre-approved configurations or be designed by the Contractor to meet OSHA standards. The contractor should submit details and calculations of any non-standard shoring or excavation support systems to the Owner prior to construction. The shoring system should prevent unacceptable movement or settlement of adjacent structures.

4.3 FOUNDATIONS

The proposed box culvert will be supported by a concrete mat foundation (i.e. the bottom of the RCB) and may be designed for an allowable soil bearing pressure of 1,500 psf.

The proposed culvert headwalls may be supported by spread footings and may be designed for an allowable soil bearing pressure of 1,500 psf. The allowable soil bearing pressure was calculated using a minimum foundation width of 12 inches and a minimum embedment depth of 18 inches. The allowable bearing pressure value may be increased by one-third for short-term loading conditions, including temporary wind and seismic forces. The allowable bearing pressure is a net value; therefore, the weight of the foundation and the weight of backfill below the lowest grade adjacent to the structure may be neglected when computing dead loads.

Resistance to lateral loads may be calculated using an allowable passive equivalent fluid unit weight as described in Lateral Earth Pressures, Section 4.2.3. Both passive and frictional resistances may be assumed to act concurrently.

We estimate that total-post construction settlement of the box culvert, designed and constructed in accordance with our recommendations will be on the order of ½ inch or less, with approximate differential settlement of on the order of ¼ inch or less along the box culvert. This assumes that new fill or other temporary or permanent surcharge load will not be placed substantially above the existing grade, including at both ends of the culvert.

4.4 UPLIFT PRESSURES

All buried structures proposed to extend below the groundwater table are subjected to uplift pressures or buoyant forces. All structures extending below the groundwater table should be designed to resist these uplift pressures. Buoyant pressures can be found by multiplying the unit weight of water (62.4 pcf) by the depth below the groundwater table. For example if the bottom of the culvert was 10 feet below the design groundwater surface, a pressure of 624 psf would be applied across the bottom of the culvert.

4.5 LATERAL EARTH PRESSURES

Lateral earth pressures will be imposed on all subterranean structures, including culverts and foundations. Table 3 and Table 4 present a list of lateral earth pressures without and with hydrostatic pressures, respectively, which we recommend for design and planning of structures. These values assume a level backfill. The values assume a minimum internal angle of friction of 30 degrees for imported or on-site granular, backfill material meeting the structural fill specification (section 4.2.5), and a unit weight of 125 pcf.

The lateral "at-rest" earth pressures should be used for design of the box culvert and headwalls. Lateral earth pressures acting against buried/retaining structures may be computed from the equivalent fluid densities presented below for the static case. The "active" condition may be used for walls that are able to deflect away from the backfill (i.e., unbraced walls). For walls that are not allowed to deflect, the "at-rest" condition should be used. The "passive" condition applies to walls or structures that move into the backfill. The uppermost 2 feet of the backfill should not be used for calculation of passive soil resistance unless it is protected by a permanent surface covering (pavement, slab, etc.). Maximum fluctuations in groundwater levels should be considered in the design.

TABLE 3
LATERAL EARTH PRESSURES
WITHOUT HYDROSTATIC PRESSURES

Earth Pressure	Equivalent Fluid Density
Active	45
Active (sloped backfill)	75
At-rest	65
Passive	375
Friction Coefficient	0.36

TABLE 4
LATERAL EARTH PRESSURES
WITH HYDROSTATIC PRESSURES

Earth Pressure	Equivalent Fluid Density
Active	85
Active (sloped backfill)	100
At-rest	95
Passive	250
Friction Coefficient	0.36

The at-rest case is applicable for braced walls where rotational movement is confined to less than 0.001H. If greater movement is possible, the active case applies.

The lateral loads computed using the values in Tables 3 and 4 assume a level backfill and granular backfill material will extend laterally at least one-half of the wall height, with the exception of the active earth pressures with sloped backfill, which have assumed a 2H:1V back slope. The sloped backfill values should be used for design of the headwalls due to the sloping roadway embankment. Non sloped backfill loads in Tables 7 and 8 may be used for all other sections of the box culvert. If this condition does not apply, the design values may require revision. This backfill should be compacted to 90% of maximum dry density and within 2% of the optimum moisture content as determined by NDOT T108. Over-compaction should be avoided, as the increased compactive effort will result in lateral pressures higher than those recommended above. Heavy compaction equipment or other loads should not be allowed in close proximity to the wall unless planned for in the structural design.

Recommended minimum factors of safety against sliding, overturning, and bearing failure are listed in Table 5, below.

TABLE 5
RECOMMENDED MINIMUM FACTORS OF SAFETY FOR RETAINING WALLS

Factor of safety against sliding	1.5
Factor of safety against overturning	2
Factor of safety against bearing failure	3*

* Factor of safety included in provided allowable bearing pressure.

If both passive and frictional resistances are assumed to act concurrently, we recommend a minimum safety factor of 2 be used for design against sliding.

4.6 CONSTRUCTION DEWATERING

Groundwater is expected to be encountered in excavations. Fluctuations in the level of the groundwater and soil moisture conditions may occur due to variations in precipitation, land use, irrigation, snow melt, river levels, and other factors.

Groundwater should ideally be lowered several feet below the bottom of the excavations to provide a firm, unyielding subgrade for construction and prevent unstable excavation wall conditions. The dewatering system should be a Contractor-designed system. Control of groundwater should be accomplished in such a manner that will preserve the strength of the foundation of soils, not cause instability of any excavated slopes or the nearby existing slopes, and not result in damage to existing structures. The water should be lowered in advance of any excavations by deep wells, well points, or other methods. Water should not be allowed to pool and remain in the excavated area over an extended period of time.

Discharge should be arranged to meet the necessary local governmental requirements and permits and to facilitate sampling by the engineer of record.

4.7 SITE DRAINAGE

Final surface grades should be designed so as to direct runoff water away from the proposed improvements and should not allow ponding. Reconstructed pavement areas should be sloped and drainage gradients maintained to match the existing conditions and carry all surface water off the site.

5 LIMITATIONS

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. The scope of services was limited to two shallow borings based on the site development plan and limited access due to utilities. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated building loads, and the design depths or locations of the foundations changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of structural fill
- Confirm structural fill for the structure and other improvements is placed and compacted per the project specifications; and
- Observe foundation bearing soils to confirm conditions are as anticipated.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of structural fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

It is the Client's responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinions, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site. The scope of our services does not include services related to construction safety precautions and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures except as specifically described in our report for consideration in design.

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than three (3) years from the date of the report. Non-compliance with any of these requirements by the client or anyone else, unless specifically agreed to in advance by Kleinfelder in writing, will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

6 REFERENCES

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PLATES

1,000 0 500 1,000

SCALE: 1 inch = 1,000 feet

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PROJECT NO.	135118
DRAWN:	07/30/2013
DRAWN BY:	D. Ross
CHECKED BY:	
FILE NAME: 135118_1.dwg	

SITE VICINITY MAP

Martin Slough - 395 Culvert Replacement
Minden, Nevada

PLATE
1

200 0 100 200

SCALE: 1 inch = 200 feet

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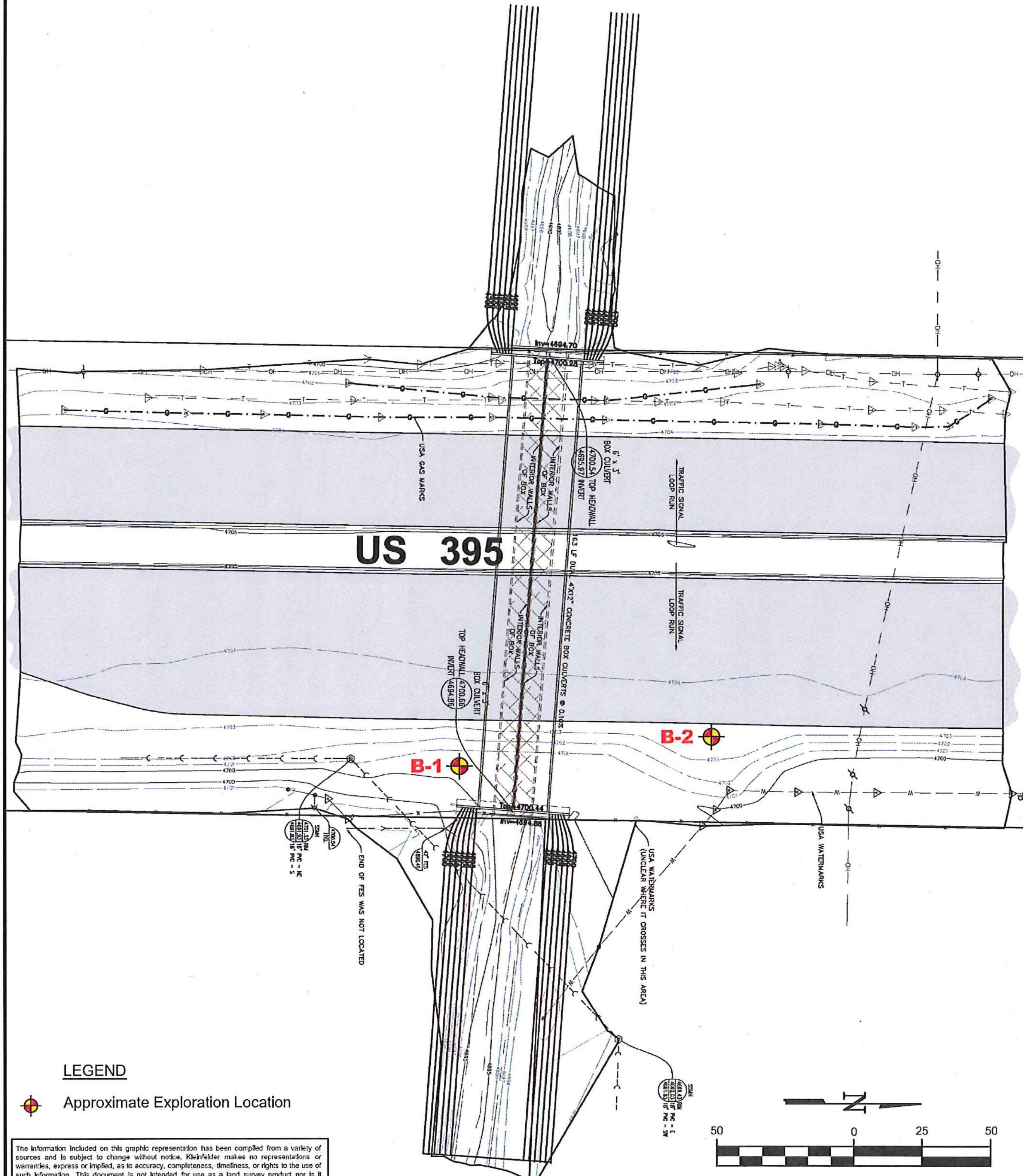
PROJECT NO.	135118
DRAWN:	07/30/2013
DRAWN BY:	D. Ross
CHECKED BY:	
FILE NAME: 135118_2.dwg	

EXPLORATION LOCATION MAP

Martin Slough - 395 Culvert Replacement
Minden, Nevada

PLATE
2



LEGEND

Approximate Exploration Location

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SCALE: 1 inch = 50 feet

Reference: GRADING PLAN, Sheet 4 of 12 by Manhard Consulting LTD, dated 07/30/2013.

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 135118 DRAWN: 07/30/2013 DRAWN BY: D. Ross CHECKED BY: FILE NAME: 135118_3.dwg	**PROPOSED CULVERT REPLACEMENT ALIGNMENT** Martin Slough - 395 Culvert Replacement Minden, Nevada	PLATE **3**

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APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

MAJOR DIVISIONS				GRAPHIC LOG		TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <5% FINES	Cu≥4 and 1≤Cc≤3		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/or 1>Cc>3		GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 to 12% FINES	Cu≥4 and 1≤Cc≤3		GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
					GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/or 1>Cc>3		GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
					GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES			GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
					GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
					GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3		SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/or 1>Cc>3		SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5 to 12% FINES	Cu≥6 and 1≤Cc≤3		SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
					SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu<6 and/or 1>Cc>3		SP-SM	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
					SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES			SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
					SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
					SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)				ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY,
					CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
					CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
					OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)				MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
					CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
					OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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Project Number: 135118	**UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)**	**Plate**
Date: 07-30-13		
Entry By: D. Ross	**Geotechnical Evaluation Minden Sough Culvert Replacement Minden, Nevada**	**A-1**
Checked By:		
File Name: 135118-1		

SOIL DESCRIPTION KEY

MOISTURE CONTENT

DESCRIPTION	ABBR	FIELD TEST
Dry	D	Absence of moisture, dusty, dry to the touch
Moist	M	Damp but no visible water
Wet	W	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

PLASTICITY

DESCRIPTION	ABBR	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm) thread cannot be rolled at any water content.
Low (L)	LP	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	MP	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	HP	It takes considerable time rolling and kneeding to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

STRUCTURE

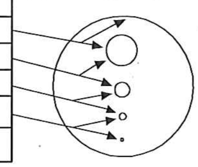
DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4 in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4 in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	ABBR	FIELD TEST
Very Soft	VS	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	S	Thumb will penetrate soil about 1 in. (25 mm)
Firm	F	Thumb will indent soil about 1/4 in. (6 mm)
Hard	H	Thumb wil not indent soil but readily indented with thumbnail
Very Hard	VH	Thumbnail will not indent soil

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12'	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3"	3/4 -3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029	Flour-sized and smaller

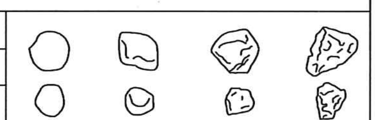


REACTION WITH HCL

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

ANGULARITY

DESCRIPTION	ABBR	CRITERIA
Angular	A	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	SA	Particles are similar to angular description but have rounded edges
Subrounded	SR	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	R	Particles have smoothly curved sides and no edges



Rounded Subrounded Subangular Angular

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	ABBR	SPT (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	VL	<4	<4	<5	0 - 15	Easily penetrated with 1/2-inch reinforcing rod by hand
Loose	L	4 - 10	5 - 12	5 - 15	15 - 35	Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand
Medium Dense	MD	10 - 30	12- 35	15 - 40	35 - 65	Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Dense	D	30 - 50	35 - 60	40 - 70	65 - 85	Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Very Dense	VD	>50	>60	>70	85 - 100	Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer

LOG SYMBOLS

BULK / BAG SAMPLE	
MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)	
CALIFORNIA SAMPLER (3 inch outside diameter)	
STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	
SHELBY TUBE	
WATER LEVEL (level where first encountered)	
WATER LEVEL (level after completion)	
SEEPAGE	

-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
LL	LIQUID LIMIT (ASTM Test Method D 4318)
PI	PLASTICITY INDEX (ASTM Test Method D 4318)
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION ASTM TEST METHOD D 4767
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION ASTM TEST METHOD D 2850
EI	EXPANSION INDEX (UBC STANDARD 18-2)
COL	COLLAPSE POTENTIAL
UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

Boring log data represents a data snapshot.

This data represents subsurface characteristics only to the extent encountered at the location of the boring.

The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities.

Lines between soil layers and/or rock units are approximate and may be gradual transitions.

The information provided should be used only for the purposes intended as described in the accompanying documents.

In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods.

Where laboratory tests were performed, the designations reflect the laboratory test results.

KA LOG KEY KA CORPORATE STD.GDT KA CORPORATE STD - 102012 - SAFCA.GLB 135118-1.GPJ 8/20/13

KLEINFELDER Bright People. Right Solutions.	**Project Number:** 135118	**LOG KEY**	**Plate**
	Date: 07-30-13		
	Entry By: D. Ross	**Geotechnical Evaluation** **Minden Sough Culvert Replacement** **Minden, Nevada**	**A-3**
	Checked By:		
	File Name: 135118-1		

Boring Number: B-1	**Boring Location:** South of culvert, 10' off fence	**Drilling Method:** Hollow-stem auger
Boring Total Depth: 9.0 ft	**Coordinates (X/Y, Lat/Long):** ft / ft	**Drilling Equipment:** CME 55
Weather:	**Datum/Coordinate System:**	**Drilling Company:** Andresen Drilling
Date Begin/End: 07-25-13 / 07-25-13	**Top of Boring Elevation:** 4700.0 ft	**Bit Size/Type:** 6 inch
Surface Conditions: Gravel/Silt	**Coordinate Data Source:** Grading Plan	**Hammer Type/Method:** Cathead
Logged By: K. Lenehan	**Depth to Groundwater Initial:** 5 feet	**Hammer Drop/Weight:** 30 in. / 140 lbs.
Field Log Reviewer:	**Depth to Groundwater Final:** 3.5 feet	**Sampler Type(s):**

The report and log key are an integral part of these logs. All data and interpretations in this log are subject to those stated explanations and limitations.

Elevation (ft)	Depth (ft)	Graphic Log	Field Soil Description and Classification	Sample Type Symbol	Sample Number	Blows per 6 in. / *Blows per ft.*	N_{60} (ASTM)	Pocket Pen (tsf)	Recovery %	Water Content % (Laboratory Data)	Dry Unit Weight (pcf) (Laboratory Data)	Liquid Limit (Laboratory Data)	Plasticity Index (Laboratory Data)	Passing #200 Sieve (%) (Laboratory Data)	Other Tests and Field Notes
			LEAN CLAY (CL): dark gray; moist; soft; 95% fines; low plasticity; 5% fine sand												
					1	4 3 7 *10*		1.0	67						
4695	5		**SANDY LEAN CLAY** (CL): dark gray; moist; very soft; 70% fines; no plasticity; 30% fine sand		2b 2a	2 3 4 *7*		0	61	34	88			43	Consolidation Test
			Poorly Graded GRAVEL With Sand (GP): dark gray; wet; dense; 35% fine to coarse sand; 65% gravel		3	14 16 13 *29*			33						
			Boring terminated at a depth of 9.0 ft below existing site grade due to heaving sand.												
4690	10														
4685	15														
4680	20														

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Project Number: 135118	**LOG OF BORING B-1**	**Plate** 1 of 1	
Date: 07-30-13		**A-4**	
Entry By: D. Ross	**Geotechnical Evaluation** **Minden Sough Culvert Replacement** **Minden, Nevada**		
Checked By: D. Adams			
File Name: 135118-1			

Boring Number: B-2	**Boring Location:** North of culvert, near roadway	**Drilling Method:** Hollow-stem auger
Boring Total Depth: 20.0 ft	**Coordinates (X/Y, Lat/Long):** ft / ft	**Drilling Equipment:** CME 55
Weather:	**Datum/Coordinate System:**	**Drilling Company:** Andresen Drilling
Date Begin/End: 07-25-13 / 07-25-13	**Top of Boring Elevation:** 4703.0 ft	**Bit Size/Type:** 6 inch
Surface Conditions: Gravel	**Coordinate Data Source:** Grading Plan	**Hammer Type/Method:** Cathead
Logged By: K. Lenehan	**Depth to Groundwater Initial:** 7.5 feet	**Hammer Drop/Weight:** 30 in. / 140 lbs.
Field Log Reviewer:	**Depth to Groundwater Final:**	**Sampler Type(s):**

The report and log key are an integral part of these logs. All data and interpretations in this log are subject to those stated explanations and limitations.

Elevation (ft)	Depth (ft)	Graphic Log	Field Soil Description and Classification	Sample Type Symbol	Sample Number	Blows per 6 in. / *Blows per ft*	N_{60} (ASTM)	Pocket Pen (tsf)	Recovery %	Laboratory Data: Water Content %	Laboratory Data: Dry Unit Weight (pcf)	Laboratory Data: Liquid Limit	Laboratory Data: Plasticity Index	Laboratory Data: Passing #200 Sieve (%)	Other Tests and Field Notes
			SILTY SAND (SM): yellowish brown; moist; dense; 25% fines; 75% medium to coarse sand; [Fill]												
4700					1	8 15 24 *39*			67					19	Grain Size
	5														
			SANDY LEAN CLAY (CL): gray; moist; hard; 70% fines; low plasticity; 30% fine sand		2b 2a	20 26 21 *47*		>4.5	56	24	94				
4695			**CLAYEY SAND** (SC): dark brown; wet; loose; 35% fines; 65% fine to medium sand		3	2 2 3 *5*			67						
	10		**LEAN CLAY** (CL): olive brown; wet; very soft; 95% fines; 5% fine sand		4b 4a	2 4 4 *8*		0.25	61	41	81	39	14		Consolidation Test
4690															
	15		**Poorly Graded GRAVEL With Sand** (GP): olive brown; wet; dense; 40% medium to coarse sand; 60% gravel		5	5 23 29 *52*			33						
4685															
	20		Boring terminated at a depth of 20.0 ft below existing site grade due to heaving sand.												

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Project Number: 135118	**LOG OF BORING B-2**	**Plate** 1 of 1
Date: 07-30-13		
Entry By: D. Ross	**Geotechnical Evaluation** **Minden Sough Culvert Replacement** **Minden, Nevada**	**A-5**
Checked By: D. Adams		
File Name: 135118-1		

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APPENDIX B

BORING NO.	SAMPLE DEPTH (ft)	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (% of dry weight)	PARTICLE SIZE SIEVE SIZE (percent passing)						ATTERBERG LIMITS		OTHER TESTS
				6"	3"	3/4"	#4	#10	#200	L.L.	P.I.	
B-1	5.5	88	34						43			
B-1	6.0											Consolidation Test
B-2	2.5					100	88	78	19			Grain Size
B-2	5.5	94	24									
B-2	11.0									39	14	Consolidation Test

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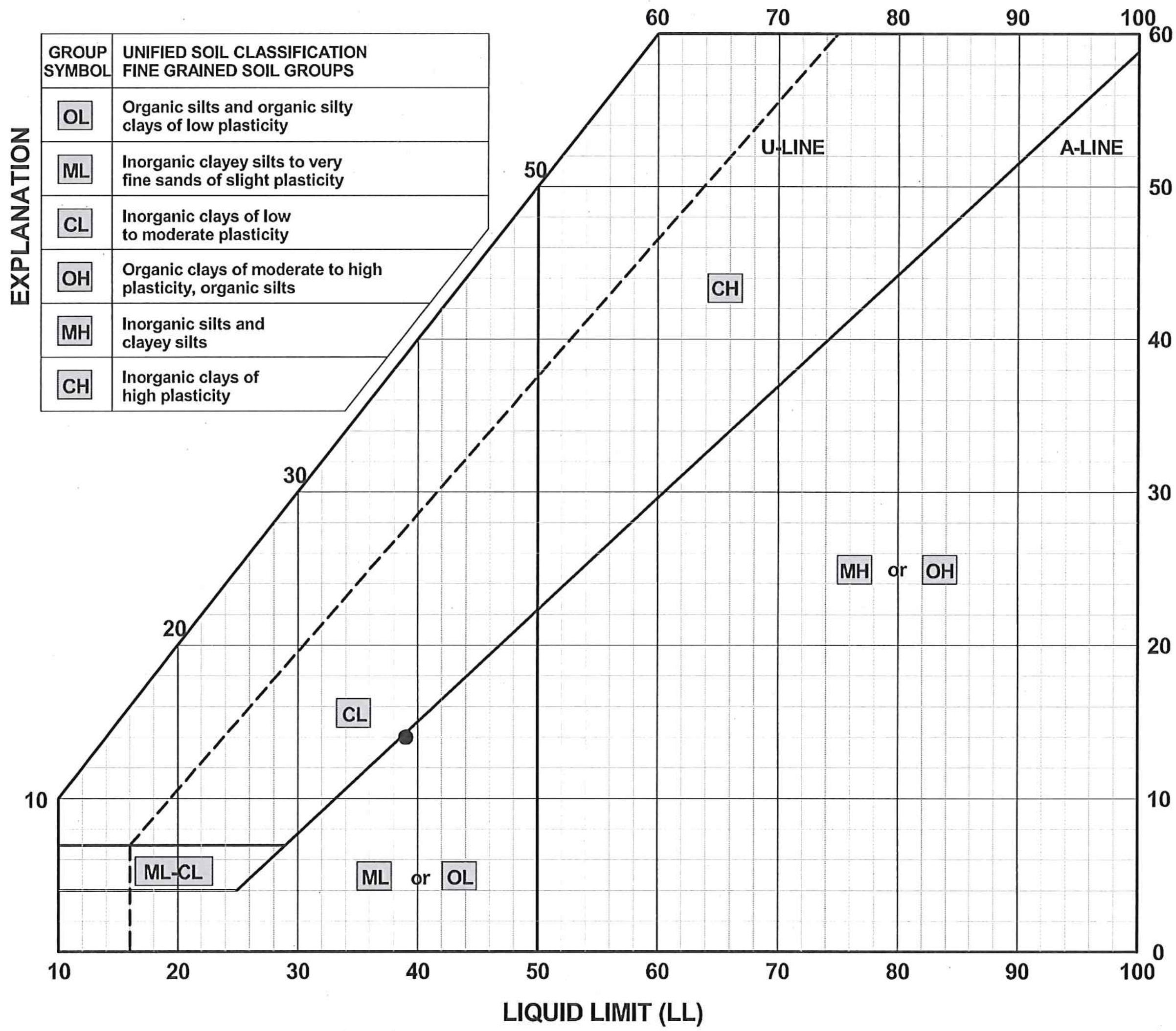
Project Number: 135118
Date: 07-30-13
Entry By: D. Ross
Checked By: D. Adams
File Name: 135118-1

SUMMARY OF LABORATORY TESTS

Geotechnical Evaluation
Minden Sough Culvert Replacement
Minden, Nevada

Plate
1 of 1
B-1

KA LAB SUMMARY KA CORPORATE STD.GDT KA CORPORATE STD - 102012 - SAFCA.GLB 135118-1.GPJ 8/20/13



LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
●	B-2	11.0	39	25	14	LEAN CLAY (CL)

KA_ATTERBERG KA_2005.GDT KA CORPORATE STD - 102012 - SAFCA.GLB 135118-1.GPJ 8/20/13

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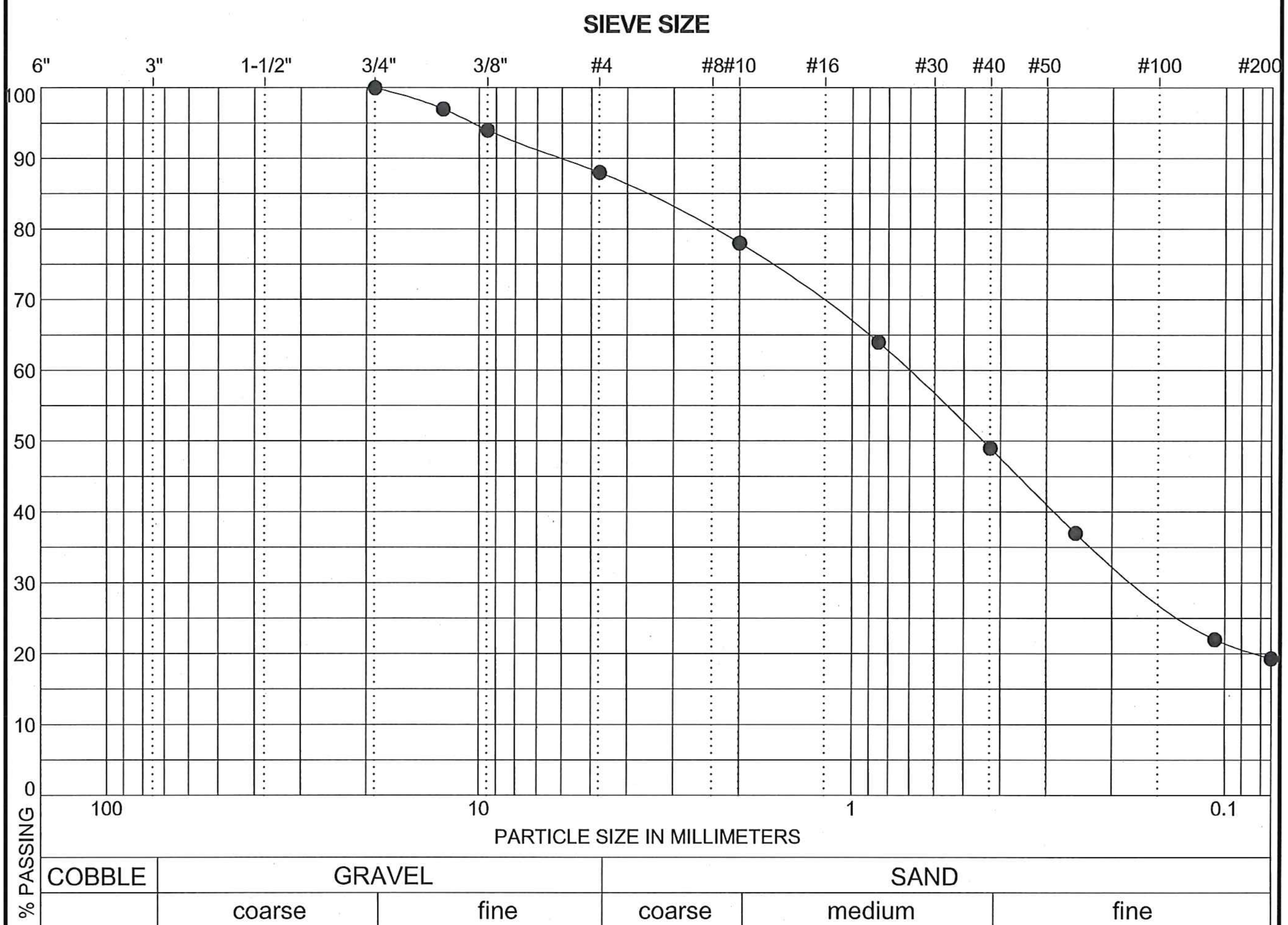
Project Number: 135118	**PLASTICITY CHART**	Plate 1 of 1
Date: 07-30-13		
Entry By: D. Ross	Geotechnical Evaluation Minden Sough Culvert Replacement Minden, Nevada	**B-2**
Checked By: D. Adams		
File Name: 135118-1		



LEGEND:	SOURCE	DEPTH (ft)	COBBLE (%)	GRAVEL (%)	SAND (%)	FINES (%)	D60 (mm)	D10 (mm)	Cu	Cc	DESCRIPTION
●	B-2	2.5	0	12	69	19	0.71				LEAN CLAY (CL)

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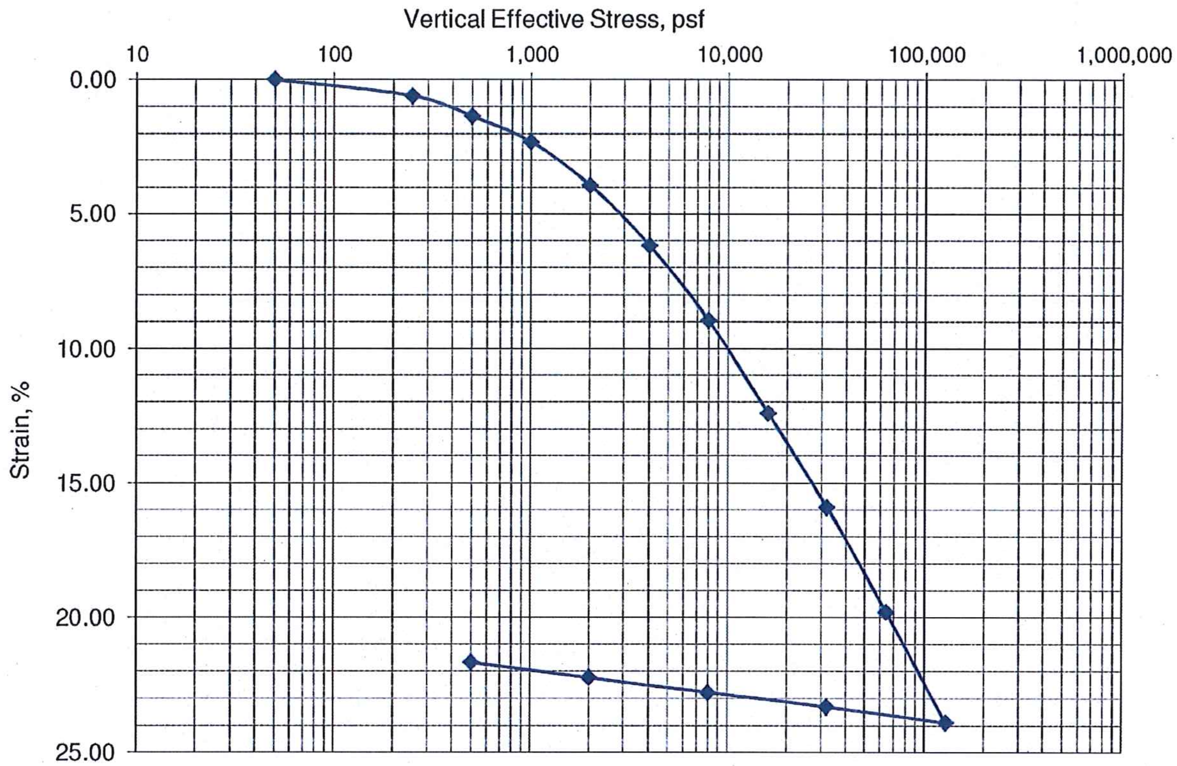
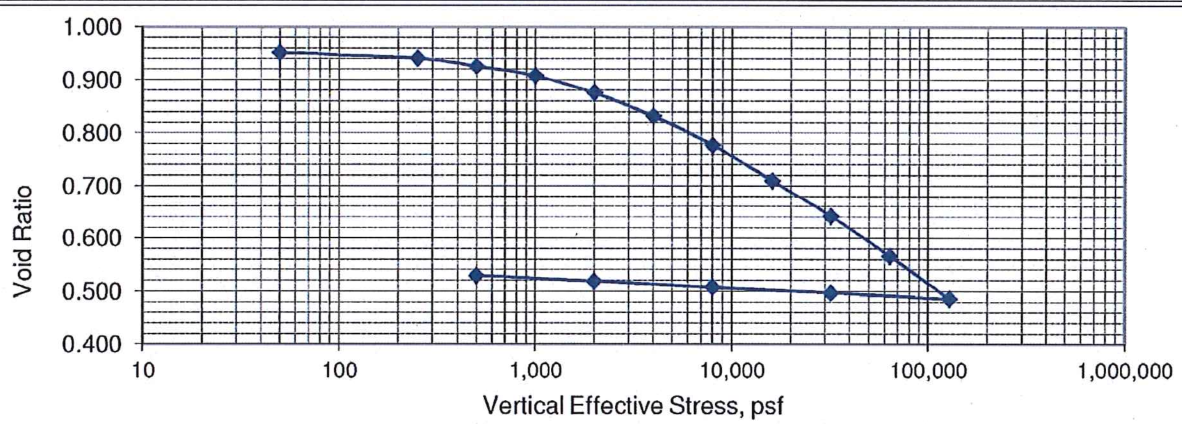
Project Number: 135118	GRAIN SIZE ANALYSIS	Plate 1 of 1	
Date: 07-30-13			
Entry By: D. Ross	Geotechnical Evaluation Minden Sough Culvert Replacement Minden, Nevada	B-3	
Checked By: D. Adams			
File Name: 135118-1			





Test Method: ASTM D2435		Sample Type: Intact		Sample Description: Silty Sand		
Gs: 2.68 Assumed	LL: nm	PI: nm	Amount of Material Finer than the No. 200, %: nm			

	Height, in.	Diameter, in.	Water Content, %	Wet Density, lb/f^3	Dry Density, lb/f^3	Saturation, %	Void Ratio
Initial	0.750	2.000	35.7	116.3	85.7	100.0	0.952
Final	0.588	2.000	19.5	131.6	110.1	100.0	0.529

Boring:	B-1	Remarks:
Sample:	2A	
Depth, ft:	6	
Test Date:	8/1/2013 - 8/17/2013	

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Project Number: 135118	**CONSOLIDATION TEST**	Plate
Date: 08/20/2013		1 of 2
Entry By: D. Ross	Geotechnical Evaluation Minden Sough Culvert Replacement Minden, Nevada	**B-4**
Checked By:		
File Name: 135118_C1.dwg		

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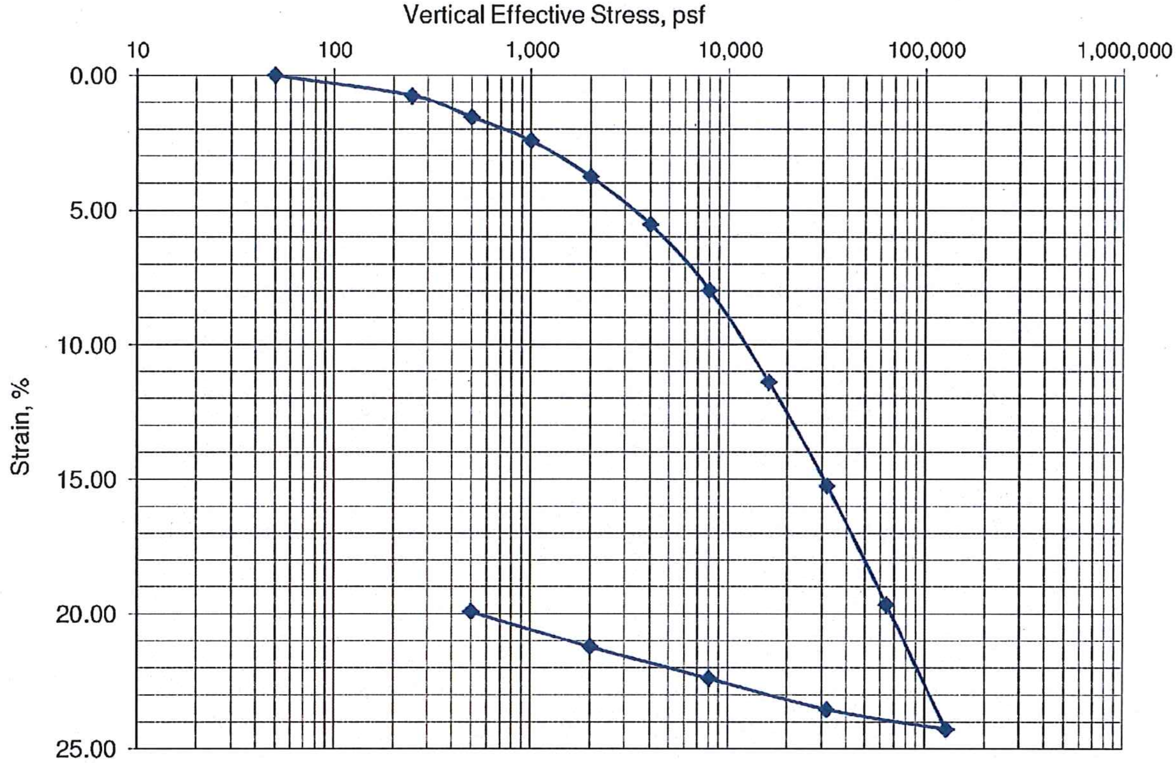

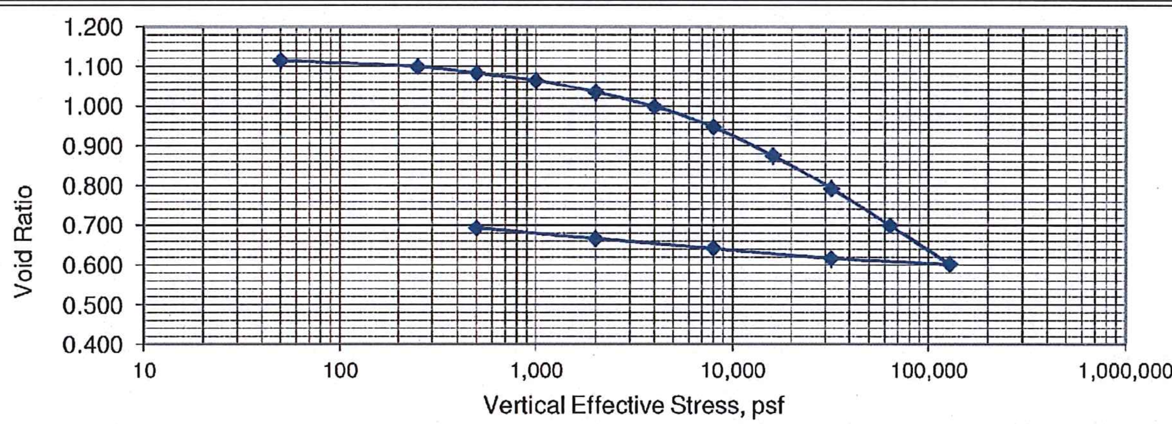

Test Method:	ASTM D2435	Sample Type: Intact		Sample Description: Lean Clay	
Gs: 2.73	Assumed	LL: 39	PI: 14	Amount of Material Finer than the No. 200, %: nm	

	Height, in.	Diameter, in.	Water Content, %	Wet Density, lb/f^3	Dry Density, lb/f^3	Saturation, %	Void Ratio
Initial	0.750	2.000	41.0	113.6	80.6	100.0	1.115
Final	0.601	2.000	25.2	126.3	100.9	100.0	0.693

Boring:	B-2	Remarks:
Sample:	4A	
Depth, ft:	11	
Test Date:	8/1/2013 - 8/17/2013	

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Project Number: 135118	**CONSOLIDATION TEST**	Plate
Date: 08/20/2013		2 of 2
Entry By: D. Ross	Geotechnical Evaluation Minden Sough Culvert Replacement Minden, Nevada	**B-4**
Checked By:		
File Name: 135118_C2.dwg		

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APPENDIX C

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
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